Chapter 3 Loads — Contents

Chapter 3 Loads

3.1 Scope

AASHTO Load and Resistance Factor Design (LRFD) Specifications shall be the minimum design criteria used for all bridges except as modified herein.

3.2 Definitions

The definitions in this section supplement those given in LRFD Section 3.

Permanent Loads – Loads and forces that are, or are assumed to be, either constant upon completion of construction or varying only over a long time interval.

Transient Loads – Loads and forces that can vary over a short time interval relative to the lifetime of the structure.

3.3 Load Designations

Load designations follow LRFD Article 3.3.2 with the addition of:

PS = secondary forces from post-tensioning

3.4 Limit States

The basic limit state equation is as follows:

$$\Sigma\eta_i\gamma_i\, Q_i \leq \phi R_n$$

where:

η_i = Limit State load modifier factor for ductility, redundancy, and importance of structure
γ_i = Load factor
Q_i = Load (i.e., dead load, live load, seismic load, etc.)
ϕ = Resistance factor
R_n = Nominal or ultimate resistance

This equation states that the force effects are multiplied by factors to account for uncertainty of in loading, structural ductility, operational importance, and redundancy, must be less than or equal to the available resistance multiplied by factors to account for variability and uncertainty in the materials and construction.

Use a value of 1.0 for η_i except for the design of columns when a minimum value of γ_i is appropriate. In such a case, use $\eta_i = 0.95$. Columns in seismic designs are proportioned and detailed to ensure the development of significant and visible inelastic deformations at the extreme event limit states before failure.

3.5 Load Factors and load combinations

The limit states load combinations, and load factors (γ_i used for structural design are in accordance with the LRFD, Table 3.4.1-1. and BDM Table 3.5-1. For foundation design, loads are factored after distribution through structural analysis or modeling.

										Use One of These at a Time			
Load Combination Limit State	DC DD DW EH EV ES EL **CR** **SH** **PS**	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	EQ	IC	CT	CV
Strength-I	γ_p	1.75	1.00	–	–	1.00	0.5/1.20	γ_{TG}	γ_{SE}	–	–	–	–
Strength-II	γ_p	1.35	1.00	–	–	1.00	0.5/1.20	γ_{TG}	γ_{SE}	–	–	–	–
Strength-III	γ_p	–	1.00	1.40	–	1.00	0.5/1.20	γ_{TG}	γ_{SE}	–	–	–	–
Strength-IV	γ_p	–	1.00	–	–	1.00	0.5/1.20	–	–	–	–	–	–
Strength-V	γ_p	1.35	1.00	0.40	1.00	1.00	0.5/1.20	γ_{TG}	γ_{SE}	–	–	–	v
Extreme Event-I	γ_p	γ_{EQ} =0.5	1.00	–	–	1.00	–	–	–	1.00	–	–	–
Extreme Event-II	γ_p	0.5	1.00	–	–	1.00	–	–	–	–	1.00	1.00	1.00
Service-I	1.00	1.00	1.00	0.30	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	–	–	–	–
Service-II	1.00	1.30	1.00	–	–	1.00	1.00/1.20	–	–	–	–	–	–
Service-III	1.00	0.80	1.00	–	–	1.00	1.00/1.20	γ_{TG}	γ_{SE}	–	–	–	–
Service-IV	1.00	–	1.00	0.70	–	1.00	1.00/1.20	–	1.00	–	–	–	–
Fatigue-LL, IM and CE only	–	0.75	–	–	–	–	–	–	–	–	–	–	–

Load Combinations and Load Factors
Table 3.5-1

The live load factor for Extreme Event-I Limit State load combination, γ_{EQ} as specified in the BDM Table 3.5.1 for all WSDOT bridges shall be taken equal to 0.50. The γ_{EQ} factor applies to the live load force effect obtained from the bridge live load analysis. Associated mass of live load need not be included in the dynamic analysis.

The AASHTO *LRFD Specifications* allow the live load factor in Extreme Event-I load combination, γ_{EQ}, be determined on a project specific basis. The commentary indicates that the possibility of partial live load, i.e., $\gamma_{EQ} < 1.0$, with earthquakes should be considered. The application of Turkstra's rule for combining uncorrelated loads indicates that $\gamma_{EQ} = 0.50$ is reasonable for a wide range of values of average daily truck traffic (ADTT). The NCHRP Report 489 recommends live load factor for Extreme Event-I Limit State, γ_{EQ} equal to 0.25 for all bridges. This factor shall be increased to γ_{EQ} equal to 0.50 for bridges located in main state routes and congested roads.

Since the determination of live load factor, γ_{EQ} based on ADTT or based on bridges located in congested roads could be confusing and questionable, it is decided that live load factor of γ_{EQ} equal to 0.50 to be used for all WSDOT bridges regardless the bridge location or congestion.

The load factors γ_{TG} and γ_{SE} are to be determined on a project specific basis in accordance with Articles 3.4.1 and 3.12 of the *LRFD Specifications*. Load Factors for Permanent Loads, γ_p are provided in LRFD Table 3.4.1-2.

The load factor for downdrag loads shall be as specified in the AASHTO specifications Table 3.4.1-2. The Geotechnical Report will provide the down drag force (DD). The down drag force (DD) is a load applied to the pile/shaft with the load factor specified in the Geotechnical Report. Generally, live loads (LL) are less than the down drag force and should be omitted when considering down drag forces. In other words, the live loads reduce down drag forces and are ignored for the structural design. The WSDOT BDM Section 8.6.2 provides a more in-depth discussion of Down Drag.

The Load Factors for Superimposed Deformations, γ_p are provided in Table 3.5-3.

	PS	CR, SH
Superstructure	1.0	1.0
Fixed (bottom) substructure supporting Superstructure (using I_g only)	0.5	0.5
All other substructure supporting Superstructure (using I_g or $I_{effective}$)	1.0	1.0

Load factors for Superimposed Deformations
Table 3.5-3

3.5.1 Load Factors for Substructure

Table 3.5-4 provides general guidelines for when to use the maximum or minimum shaft/pile/column permanent load factors for axial capacity, uplift, and lateral loading.

In general, substructure design should use unfactored loads to obtain force distribution in the structure, and then factor the resulting moment and shear for final structural design. All forces and load factors are as defined previously.

Axial Capacity	Uplift	Lateral Loading
DC_{max}, DW_{max}	DC_{min}, DW_{min}	DC_{max}, DW_{max}
DC_{max}, DW_{max} for causing shear	DC_{max}, DW_{max} for causing shear	DC_{max}, DW_{max} causing shear
DC_{min}, DW_{min} for resisting shear	DC_{min}, DW_{min} for resisting shear	DC_{min}, DW_{min} resisting shear
DC_{max}, DW_{max} for causing moments	DC_{max}, DW_{max} for causing moments	DC_{max}, Dw_{max} for causing moments
DC_{min}, DW_{min} for resisting moments	DC_{min}, DW_{min} for resisting moments	DC_{min}, DW_{min} for resisting moments
EV_{max}	EV_{min}	EV_{max}
DD = varies	DD = varies	DD = varies
EH_{max}	EH_{max} if causes uplift	EH_{max}

Minimum/Maximum Substructure Load Factors for Strength Limit State
Table 3.5-4

In the table above "causing moment" and "causing shear" are taken to be the moment and shear causing axial, uplift, and lateral loading respectively. "Resisting" is taking to mean those force effects that are diminishing axial capacity, uplift, and lateral loading.

3.6 Loads and Load factors for construction

Unless otherwise specified, the load factor for construction loads and for any associated dynamic effects shall not be less than 1.5 in Strength I. The load factor for wind in Strength III shall not be less than 1.25.

When investigating Strength Load Combinations I, III, and V during construction, load factors for the weight of the structure and appurtenances, *DC* and *DW*, shall not be taken to be less than 1.25.

Where evaluation of construction deflections are required by the contract documents, Load Combination Service I shall apply. Construction dead loads shall be considered as part of the permanent load and construction transient loads considered part of the live load. The associated permitted deflections shall be included in the contract documents.

For falsework and formwork design loads, see standard specifications 6-02.3(17)A.

3.7 Load factors for Post-tensioning

3.7.1 Post-tensioning Effects from Superstructure

When cast-in-place, post-tensioned superstructure is constructed monolithic with the piers, the substructure design should take into account frame moments and shears caused by elastic shortening and creep of the superstructure upon application of the axial post-tensioning force at the bridge ends. Frame moments and shears thus obtained should be added algebraically to the values obtained from the primary and secondary moment diagrams applied to the superstructure.

When cast-in-place, post-tensioned superstructure are supported on sliding bearings at some of the piers, the design of those piers should include the longitudinal force from friction on the bearings generated as the superstructure shortens during jacking. When post-tensioning is complete, the full permanent reaction from this effect should be included in the governing AASHTO load combinations for the pier under design.

3.7.2 Secondary Forces from Post-Tensioning, PS

The application of post-tenstioning forces on a continuous structure produces reactions at the structure's support and internal forces that are collectively called secondary forces.

Secondary prestressing forces (i.e. secondary moments) are the force effects in continuous members, as a result of continuous post-tensioning. In frame analysis software, the secondary moments are generally obtained by subtracting the primary ($P*e$) from the total PS moments. Whether or not this is appropriate when using linear-elastic analysis is debatable, but accepted for lack of a better method. A load factor, γ_{PS}, of 1.0 is appropriate for the superstructure. For fixed columns a 50% reduction in PS force effects could be used given the elasto-plastic characteristics of the soil surrounding the foundation elements.

3.8 Permanent Loads

The design unit weights of common permanent loads are provided in Table 3.8-1.

ITEM	LOAD
Reinforced Concrete	160 lb/ft^3
Concrete Overlay	150 lb/ ft^3
Stay-in-Place Form for Box Girder (applied to slab area less overhangs and webs)	5 lb/ft^2
Traffic Barrier (32” - F Shape)	470 lb/ft
Traffic Barrier (42” - F Shape)	730 lb/ft
Traffic Barrier (34” – Single Slope)	505 lb/ft
Traffic Barrier (42” – Single Slope)	690 lb/ft
Wearing Surface – Asphalt Concrete Pavement (ACP)	125 lb/ft^3
Wearing Surface – Hot Mix Asphalt (HMA)	140 lb/ft^3
Soil, Compact	125 lb/ft^3

Permanent Loads
Table 3.8-1

3.8.1 Deck Overlay Requirement

Vehicular traffic will generate wear and rutting on a concrete bridge deck over the life of a bridge. One option to correct excessive wear is to add a Hot Mix Asphalt (HMA) overlay on top of the existing concrete deck. This type of overlay requires less construction time and is less expensive compared to removing a portion of the deck and adding a modified concrete overlay. The initial bridge design needs to incorporate the future overlay dead load.

Concrete bridge deck protection systems shall be in accordance with the requirements of BDM Section 5.7.4 for new bridge construction and widening projects. To accommodate a future deck overlay, bridges shall be designed as shown in the following Table.

Superstructure Type	Concrete Cover	Overlay shown in the plan	Future Design Overlay
System 1: • Precast concrete, steel I or box girder with cast-in-place slab • Precast slabs with cast-in-place slab • Reinforced and post-tensioned box beams and slab bridges • Mainline Bridges on State Routes	2 ½" (Including ½" wearing surface)	None	2" HMA
System 1: • Undercrossing bridge that carries traffic from a city street or county road • Bridges with raised sidewalks	2 ½" (Including ½" wearing surface)	None	None
System 2: • Decks of segmental bridges with transverse post-tensioning	1 ¾" (Including ¼" wearing surface)	1 ½" Modified Concrete Overlay	None
System 3: • Deck bulb tees, Double tees and tri-beams	2"	3" HMA	None

Bridge Overlay Requirements
Table 3.8-2

The effect of the future deck overlay on girders camber, "A" dimension, creep, and profile grade need not be considered in superstructure design.

Deck overlay may be required at the time of original construction for some bridge widening or staged construction projects if ride quality is a major concern.

3.9 Live Loads

3.9.1 Live Load Designation

Live load design criteria are specified in the lower right corner of the bridge preliminary plan sheet. The Bridge Projects Unit determines the criteria using the following guideline:

- New bridges and Bridge widening with addition of substructure – HL-93
- Bridge superstructure widening with no addition of substructure – Live load criteria of the original design
- Detour and other temporary bridges – 75% of HL-93

3.9.2 Live Load Analysis of Continuous Bridges

The HL-93 live load model defined in the *LRFD specifications* includes a dual truck train for negative moments and reactions and interior piers. The application of the dual truck train is somewhat unclear as specified in LRFD Article 3.6.1.3.1. WSDOT interprets that article as follows:

For negative moment between the points of contraflexure under a uniform load on all spans, shear, and reactions at interior piers only, 90 percent of the effect of two design trucks spaced a minimum of 50.0 ft. between the rear axle of the lead truck and the lead axle of the rear truck, combined with 90 percent of the effect of the design lane load. The distance between the 32.0-kip axles of each truck shall be taken as 14.0 ft. The two design trucks shall be placed in different spans in such position to produce maximum force effect.

Negative moment, shear, and reactions at interior supports shall be investigated a dual design tandem spaced from 26.0 ft. to 40.0 ft apart, combined with the design lane load specified in LRFD Article C3.6.1.3.1. For the purpose of this article, the pairs of the design tandem shall be placed in different spans in such position to produce maximum force effect.

3.9.3 Loading for Live Load Deflection Evaluation

The loading for live load deflection criteria is defined in LRFD Article 3.6.1.3.2. Live load deflections for the Service I limit state shall satisfy the requirements of LRFD 2.5.2.6.2.

3.9.4 Distribution to Superstructure

A. Multi Girder Superstructure

The live load distribution factor for exterior girder of multi girder bridges shall be as follows:

- For exterior girder design with slab cantilever length equal or less than one-half of the adjacent interior girder spacing, use the live load distribution factor for interior girder. The slab cantilever length is defined as the distance from the centerline of the exterior girder to the edge of the slab.
- For exterior girder design with slab cantilever length exceeding one-half of the adjacent interior girder spacing, use the lever rule with the multiple presence factor of 1.0 for single lane to determine the live load distribution. The live load used to design the exterior girder shall not be less than the live load used for the adjacent interior girder.
- The special analysis based on the conventional approximation of loads on piles as described in LRFD Article C4.6.2.2.2d shall not be used unless the effectiveness of diaphragms on the lateral distribution of truck load is investigated.

B. Concrete Box Girders

The load distribution factor for multi-cell cast in place concrete box girders shall be per *LRFD Specifications* for interior girders from Table 4.6.2.2.2b-1 for bending moment, and Table 4.6.2.2.3a-1 for shear. The live load distribution factor for interior girders shall then be multiplied by the number of webs to obtain the design live load for the entire superstructure. The correction factor for live load distribution for skewed support as specified in Tables 4.6.2.2.2e-1 for bending moment and 4.6.2.2.3c-1 for shear shall be considered.

$DF = N_b \times Df_i$ Live load distribution factor for multi-cell box girder

Where:
Df_i = Live load distribution factor for interior web
N_b = Number of webs

C. Multiple Presence Factors

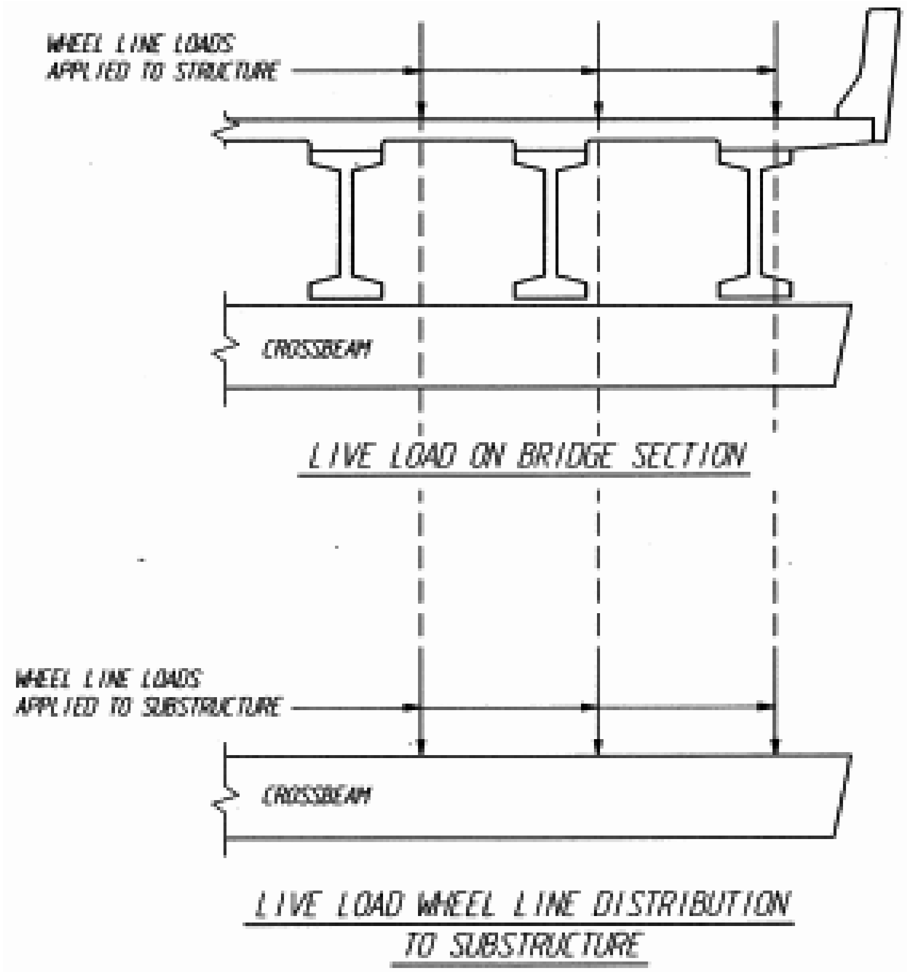
A reduction factor will be applied in the substructure design for multiple loadings in accordance with AASHTO.

D. Distribution to Substructure

The number of traffic lanes to be used in the substructure design shall be determined by dividing the entire roadway slab width by 12. No fractional lanes shall be used. Roadway slab widths of less than 24 feet shall have a maximum of two design lanes.

E. Distribution to Crossbeam

The HL-93 loading is distributed to the substructure by placing wheel line reactions in a lane configuration that generates the maximum stress in the substructure. A wheel line reaction is ½ of the HL-93 reaction. Live loads are considered to act directly on the substructure without further distribution through the superstructure as illustrated in Table 3.9-1. Normally, substructure design will not consider live load torsion or lateral distribution. Sidesway effects may be accounted for and are generally included in computer generated frame analysis results.



Live Load Distribution to Substructure
Table 3.9-1

For steel and prestressed concrete superstructure where the live load is transferred to substructure through bearings, cross frames or diaphragms, the girder reaction may be used for substructure design. Live load placement is dependant on the member under design. Some examples of live load placement are as follows. The exterior vehicle wheel is placed 2 feet from the curb for maximum crossbeam cantilever moment or maximum eccentric foundation moment.

For crossbeam design between supports, the HL-93 lanes are placed to obtain the maximum moment in the member; then re-located to obtain the maximum shear or negative moment in the member.

For column design, the design lanes are placed to obtain the maximum transverse moment at the top of the column; then re-located to obtain the maximum axial force of the column.

3.10 Pedestrian Loads

A pedestrian load of 0.075 ksf shall be applied to all sidewalks wider than 2.0 ft and considered simultaneously with the vehicular design live load. For purposes of determining the number of lanes loaded when combined with one or more lanes of vehicular live load, the pedestrian loads may be taken to be one loaded lane.

Bridges for only pedestrian and/or bicycle traffic shall be designed for a live load of 0.085 ksf.

Where sidewalks, pedestrian, and/or bicycle bridges are intended to be used by maintenance and/or other incidental vehicles, these loads shall be considered in the design. The dynamic load allowance need not be considered for these vehicles and shall not be considered concurrently with the pedestrian load.

The maintenance vehicle live load shall be:

Sidewalk Width	**Truck**
Less than 6 ft	N/A
6ft – 10ft	H-5
Greater than 10ft	H-10 UBIT Load (consult the BPO engineer for details)

When a future bridge widening is anticipated, the exterior girders shall be designed with the sidewalk removed and the full live load considered.

3.11 Wind Loads

3.11.1 Wind Load to Superstructure

For the usual girder and slab bridges with less than 30 ft height above ground, the following simplified wind pressure on structure (WS), could be used in lieu of the general method described in LRFD Article 3.8.1.2:

- 0.05 kip per square foot, transverse
- 0.012 kip per square foot, longitudinal

Both forces shall be applied simultaneously.

For the usual girder and slab bridges with less than 30 ft height above ground, the following simplified wind pressure on vehicle (WL), could be used in lieu of the general method described in LRFD Article 3.8.1.3:

- 0.10 kip per linear foot, transverse
- 0.04 kip per linear foot, longitudinal

Both forces shall be applied simultaneously.

3.11.2 Wind Load to Substructure

Wind forces shall be applied to the substructure units in accordance with the loadings specified in AASHTO. Transverse stiffness of the superstructure may be considered, as necessary, to properly distribute loads to the substructure provided that the superstructure is capable of sustaining such loads. Vertical wind pressure, per LRFD 3.8.2, shall be included in the design where appropriate, for example, on single column piers. Wind loads shall be applied through shear keys or other positive means from the superstructure to the substructure. Wind loads shall be distributed to the piers and abutments in accordance with the laws of statics. Transverse wind loads can be applied directly to the piers assuming the superstructure to act as a rigid beam. For large structures a more appropriate result might be obtained by considering the superstructure to act as a flexible beam on elastic supports.

3.11.3 Wind on Noise Walls

Wind load shall be assumed to be uniformly distributed on the area exposed to the wind, taken perpendicular to the assumed wind direction. Design wind pressure may be determined using either the tabulated values given below or the design equations that follow.

Height of structure, Z, at which wind loads are being calculated as measured from low ground, or water level.	Wind Velocity (mph)		
	80 mph	90 mph	100 mph
0 - 30 ft.	4 psf	5 psf	6 psf
30 - 40 ft.	6 psf	7 psf	9 psf
40 - 50 ft.	8 psf	10 psf	12 psf

Minimum Wind Pressure for City Terrain (Exposure A)
Table 3.11-1

Height of structure, Z, at which wind loads are being calculated as measured from low ground, or water level.	Wind Velocity (mph)		
	80 mph	**90 mph**	**100 mph**
0 - 30 ft.	9 psf	12 psf	15 psf
30 - 40 ft.	12 psf	15 psf	19 psf
40 - 50 ft.	14 psf	18 psf	22 psf

Minimum Wind Pressure for Suburban Terrain (Exposure B1)
Table 3.11-2

Height of structure, Z, at which wind loads are being calculated as measured from low ground, or water level.	Wind Velocity (mph)		
	80 mph	**90 mph**	**100 mph**
0 - 30 ft.	17 psf	21 psf	26 psf
30 - 40 ft.	19 psf	25 psf	30 psf
40 - 50 ft.	22 psf	28 psf	34 psf

Minimum Wind Pressure for Sparse Suburban Terrain (Exposure B2)
Table 3.11-3

Height of structure, Z, at which wind loads are being calculated as measured from low ground, or water level.	Wind Velocity (mph)		
	80 mph	**90 mph**	**100 mph**
0 - 30 ft.	26 psf	32 psf	40 psf
30 - 40 ft.	29 psf	36 psf	45 psf
40 - 50 ft.	31 psf	39 psf	49 psf

Minimum Wind Pressure for Open Country Terrain (Exposure C)
Table 3.11-4

Height of structure, Z, at which wind loads are being calculated as measured from low ground, or water level.	Wind Velocity (mph)		
	80 mph	**90 mph**	**100 mph**
0 - 30 ft.	39 psf	50 psf	62 psf
30 - 40 ft.	43 psf	54 psf	67 psf
40 - 50 ft.	45 psf	57 psf	71 psf

Minimum Wind Pressure for Coastal Terrain (Exposure D)
Table 3.11-5

Design Wind Pressure

For noise walls with heights greater than 50 ft. or subjected to wind velocities other than 80, 90, or 100 mph, the following equations shall be used to determine the minimum design wind pressure to be applied to the wall:

$$P = P_B \left(\frac{V_{DZ}}{V_B} \right)^2$$

Where

P = design wind pressure (psf)
P_B = base wind pressure (psf)
V_{DZ} = design wind velocity at design elevation (mph)
V_B = base wind velocity (100 mph) at 30.0 ft height

Base Wind Pressure

The base wind pressure, P_B, shall be taken as 40 psf for walls and other large flat surfaces.

Design Wind Velocity

The design wind velocity is computed as:

$$V_{DZ} = 2.5V_0\left(\frac{V_{30}}{V_B}\right)\ln\left(\frac{Z}{Z_0}\right)$$

Where

V_0 = friction velocity (mph)
V_{30} = wind velocity at 30.0 ft above low ground or above design water level (mph)
Z = height of structure at which wind loads are being calculated as measured from low ground or water level, > 30.0 ft
Z_0 = friction length of upstream fetch (ft), (also referred to as roughness length)

Exposure Categories

City (A): Large city centers with at least 50 percent of the buildings having a height in excess of 70.0 ft. Use of this category shall be limited to those areas for which representative terrain prevails in the upwind direction at least one-half mile. Possible channeling effects of increased velocity pressures due to the bridge or structure's location in the wake of adjacent structures shall be accounted for.

Suburban (B1): Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family or larger dwellings. This category shall be limited to those areas for which representative terrain prevails in the upwind direction at least 1500 ft.

Sparse Suburban (B2): Urban and suburban areas with more open terrain not meeting the requirements of Exposure B1.

Open Country (C): Open terrain with scattered obstructions having heights generally less than 30.0 ft. This category includes flat open country and grasslands.

Coastal (D): Flat unobstructed areas and water surfaces directly exposed to wind. This category includes large bodies of water, smooth mud flats, salt flats, and unbroken ice.

Friction Velocity

A meteorological wind characteristic taken for various upwind surface characteristics (mph).

Condition	City	Suburbs	Sparse Suburbs	Open Country	Coastal
V_0 (mph)	12.0	10.9	9.4	8.2	7.0

Wind Velocity at 30.0 ft

V_{30} may be established from:

Fastest-mile-of-wind charts available in ASCE 7-88 for various recurrence

Site-specific wind surveys, or

In the absence of better criterion, the assumption that $V_{30} = V_B = 100$ mph.

Friction Length

A meteorological wind characteristic of upstream terrain (ft).

Condition	City	Suburbs	Sparse Suburbs	Open Country	Coastal
Z_0 (ft)	8.20	3.28	0.98	0.23	0.025

3.12 Noise Barriers

The design requirement for noise barrier wall on bridges and walls are as follows:

1. The total height of noise barrier wall on bridges, from top of slab to top of noise barrier wall, shall be limited to 8'-0"
2. The total height of noise barrier wall on retaining walls, from top of roadway to top of noise barrier wall, shall be limited to 14'-0"
3. Noise barrier wall thickness shall be 7 inches minimum
4. Two layers of reinforcing bars shall be specified in the cross section, with 1.5" cover, minimum, over both faces as shown in the attached detail.
5. Wind load shall be based on BDM Section 3.11
6. The vehicular collision force shall be based on the LRFD Table A13.2-1 for design forces for traffic railing. The transverse force shall be applied horizontally at 3'-6" height above deck.
7. Seismic load shall be as follows:

   Seismic Dead Load = A x f x D

   Where:
   A = Acceleration coefficient from the Geotechnical Report
   D = Dead load of the wall
   f = Dead load coefficient

Dead Load Coefficient, f	
Dead load coefficient, except on bridges – monolithic connection	1.0
Dead load coefficient, on bridges – monolithic connection	2.5
Dead load coefficient, for connection of precast wall to bridge barrier	8.0
Dead load coefficient, for connection of precast walls to retaining wall or moment slab barriers	5.0

   The product of A and f shall not be taken less than 0.10.
8. LRFD Bridge design specifications shall be used for the structural design of noise barrier walls.

3.13 Earthquake Effects

Earthquake Loads (see BDM Chapter 4)

3.14 Earth Pressure

Earthquake Loads (see BDM Chapter 7)

3.15 Force Effects due to superimposed deformations

PS, CR, SH, TU and TG are superimposed deformations. Load factors for PS, CR, and SH, are as shown in Table 3.5-3. In non-segmental structures: PS, CR and SH are symbolically factored by a value of 1.0 in the strength limit state, but are actually designed for in the service limit state. For substructure in the strength limit state, the value of 0.50 for γ_{PS}, γ_{CR}, γ_{SH}, and γ_{TU} may be used when calculating force effects in non-segmental structures, but shall be taken in conjunction with the gross moment of inertia in the columns or piers. The larger of the values provided for load factor of TU shall be used for deformations and the smaller values for all other effects. The calculation of displacements for TU loads utilizes a factor greater than 1.0 to avoid under sizing joints, expansion devices, and bearings.

The current *LRFD Specifications* require a load factor of 1.2 on *CR*, *SH*, and *TU* deformations, and 0.5 on other *CR/SH/TU* force effects. The lower value had been rationalized as dissipation of these force effects over time, particularly in the columns and piers.

Changing the load factors for creep and shrinkage is not straight-forward because *CR*, *SH* are "superimposed deformations", that is, force effects due to a change in material behavior that cause a change in the statical system. For safety and simplicity in design, they are treated as loads--despite not being measurable at time $t = 0$. However, behavior is nonlinear and application of the load factor must also be considered. Some software will run service load analysis twice: once with and once without *CR*, *SH* effects. The *CR* and *SH* can then be isolated by subtracting the results of the two runs. Other software will couple the *CR* and *SH* with the dead load, giving a shrinkage- or creep-adjusted dead load.

The proposed compromise is to assign creep and shrinkage the same load factor as the *DC* loads, but permit a factor of 1.0 if the project-specific creep coefficient can be determined and is then used in the linear analysis software.

Thermal and shrinkage loadings are induced by movements of the structure and can result from several sources. Movements due to temperature changes are calculated using coefficients of thermal expansion of 0.000006 ft/ft per degree for concrete and 0.0000065 ft/ft per degree for steel. Reinforced concrete shrinks at the rate of 0.0002 ft/ft.

3.16 Other Loads

3.16.1 Buoyancy

The effects of submergence of a portion of the substructure is to be calculated, both for designing piling for uplift and for realizing economy in footing design.

3.16.2 Collision Force on Bridge Substructure

See LRFD Articles 3.6.5 and 3.14

3.16.3 Collision Force on Traffic Barrier

See LRFD Article 3.6.5.1

3.16.4 Force from Stream Current, Floating Ice, and Drift

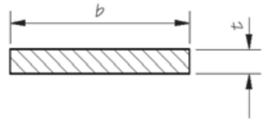
See AAHTO LRFD Article 3.9

3.16.5 Ice Load

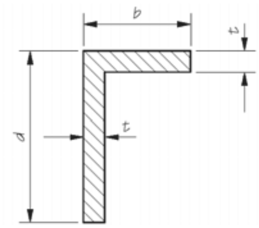
In accordance with WSDOT HQ Hydraulics Office criteria, an ice thickness of 12 inches shall be used for stream flow forces on piers throughout Washington State.

3.99 Bibliography

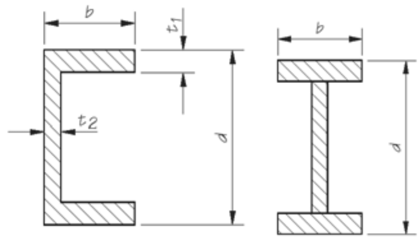
1. AASHTO, *LRFD Bridge Design Specifications for Design of Highway Bridges*, 2004 and interims through 2007.



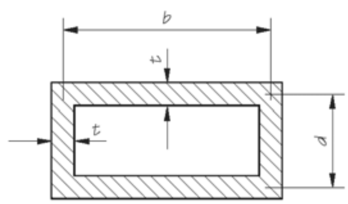
$$R = \frac{bt^3}{3}$$



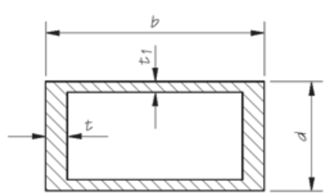
$$R = \frac{(b+d)t^3}{3}$$



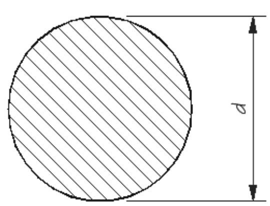
$$R = \frac{2bt_1^3 + dt_2^3}{3}$$



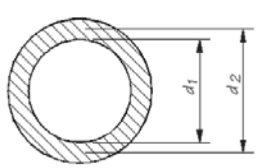
$$R = \frac{2tb^2d^2}{b+d}$$



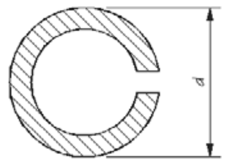
$$R = \frac{2tt_1(b-t)^2(d-t_1)^2}{bt + dt_1 - t^2 - t_1^2}$$



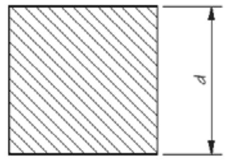
$$R = 0.0982d^4$$



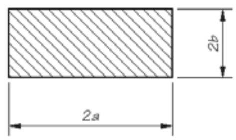
$$R = 0.0982\left(d_2^4 - d_1^4\right)$$



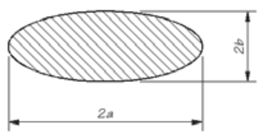
$$R = 1.0472t^3 d$$



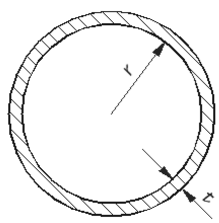
$$R = 0.1406d^4$$



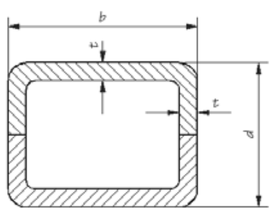
$$R = ab^3\left[\frac{16}{3} - 3.36\frac{b}{a}\left(1 - \frac{b^4}{12a^4}\right)\right]$$



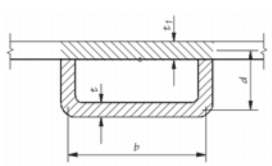
$$R = \frac{\pi a^3 b^3}{a^2 + b^2}$$



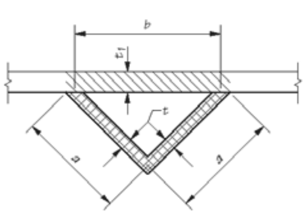
$$R = 2\pi r^3 t$$



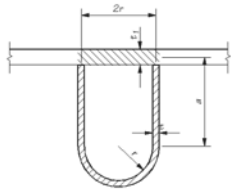
$$R = \frac{2tb^2 d^2}{b + d}$$



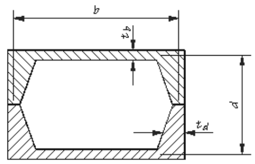
$$R = \frac{4b^2 d^2}{\dfrac{b + 2d}{t} + \dfrac{b}{t_1}}$$



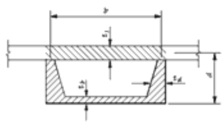
$$R = \frac{a^4}{\dfrac{2a}{t} + \dfrac{b}{t_1}}$$



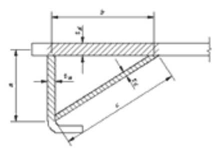
$$R = \frac{4r^2\left(\frac{\pi r}{2} + 2a\right)^2}{\frac{2a + \pi r}{t} + \frac{2r}{t_1}}$$



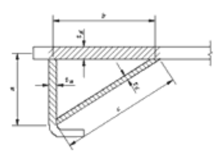
$$R = \frac{2b^2 d^2}{\frac{b}{t_b} + \frac{d}{t_d}}$$



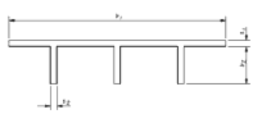
$$R = \frac{4b^2 d^2}{\frac{b}{t_b} + \frac{2d}{t_d} + \frac{b}{t_1}}$$



$$R = \frac{a^2 b^2}{\frac{a}{t_a} + \frac{b}{t_b} + \frac{c}{t_c}}$$



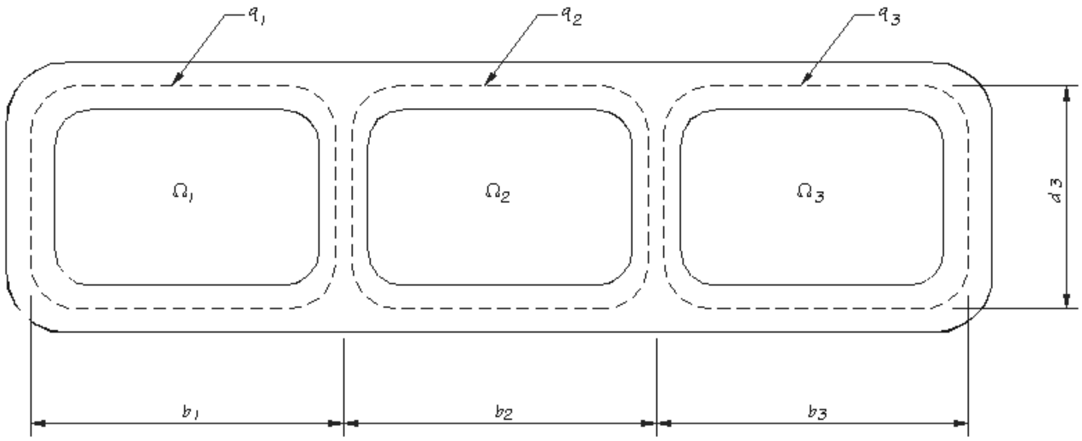
$$R = \frac{a^2 b^2}{\frac{a}{t_a} + \frac{b}{t_b} + \frac{c}{t_c}}$$



$$R = \frac{b_1 t_1^3 + 3 b_2 t_2^3}{3}$$

Multi-Celled Sections

Torsion of two or more cells connect at the walls is a statically indeterminate problem. The general method to find the torsional rigidity, R, is as follows:



The equation for equilibrium for n cells is:

$$M_t = 2\sum_{i=1}^{n} q_i \Omega_i \tag{1}$$

Where q_i is the shear flow in cell i and Ω_i is the area enclosed by the center line of the walls inclosing the cell, and M_t is the twisting moment applied to the cell.

The equations of consistent deformation are:

$$S_{ji}q_i + S_{jj}q_j + S_{jk}q_k = 2\Omega_j\theta \tag{2}$$

Where:

$$S_{ji} = -\frac{1}{G}\int S_{ji}\,\frac{ds}{t}$$

$$S_{jj} = -\frac{1}{G}\int S_{jj}\,\frac{ds}{t}$$

$$S_{jk} = -\frac{1}{G}\int S_{jk}\,\frac{ds}{t}$$

G is the shear modulus of elasticity

$\int S_{ji}\frac{ds}{t}$ is the sum of the length of cell wall, common to cells j and i, divided by its thickness

$\int S_{jk}\frac{ds}{t}$ is the sum of the length of cell wall, common to cells j and k, divided by its thickness

$\int S_{jj}\frac{ds}{t}$ is the sum of the length of cell wall, common to cell j, divided

by their respective thicknesses.

θ is the angle of twist in radians

Equation (2) will yield n equations for n unknown shear flows and can be solved for the shear flows q_i in terms for G and the angle of twist θ. Knowing θ_i and Ω_i the torsional constant R may be calculated from:

$$R = \frac{2}{G\theta}\sum_{i=1}^{n} q_i \Omega_i$$

A simplification of this method is to assume that the interior web members are not effective in torsion. The torsional constant may be approximated by:

$$R = \frac{4A^2}{\sum_i \frac{S_i}{t_i}}$$

Where:

- A is the area enclosed by the centerline of the exterior webs and the top and bottom slabs
- S_i is the length of side i
- t_i is the thickness of side i

Appendix 3.1-B1 HL-93 Loading for Bridge Piers

1. Introduction

   The purpose of this example is to demonstrate a methodology of analyzing a bridge pier for the HL-93 live load on two-dimension plane frame in both longitudinal and transverse directions. First, the longitudinal analysis of the superstructure is analyzed. This analysis produces the live load reactions at the intermediate piers. Then the reactions are applied in the transverse direction, for the crossbeam and column design.

2. Bridge Description

3. Analysis Goals

   To determine:

   - Maximum axial forces and corresponding moments.
   - Maximum moments and corresponding axial forces.
   - Maximum shears.

4. Material Properties

   4.1 Girders

   $w_c = 0.160\ kcf$
   $f'c = 7ksi$
   $E_c = 33{,}000(0.160)^{1.5}\ \sqrt{7} = 5588$ ksi

   4.2 Slab, Columns, and Cross Beam

   $wc = 0.160\ kcf$
   $f'_c = 4\ ksi$
   $E_c = 33{,}000(0.160)^{1.5}\ \sqrt{4} = 4224$ ksi

5. Section Properties

   Compute the geometric properties of the girder, columns, and cap beam.

   5.1. Girder

   The composite girder section properties can be obtained from the Section Properties Calculator in QConBridge program for the longitudinal direction.

   $A = 1254.6\ in^2$
   $I = 1{,}007{,}880\ in^3$

   5.2. Column

   Section properties of an individual column are obtained by simple formula for longitudinal and transverse directions:

   For the longitudinal analysis we need to proportion the column stiffness to each girder line. For longitudinal analysis the section properties of the each column member are:

   NOTE

   For other column shapes and columns on a skewed bent, the properties of the columns need to be computed in the plane of the longitudinal and transverse frames respectively for analysis in each direction.

5.3. Cap Beam

Cap beam properties are obtained by simple formula in transverse direction:

6. Longitudinal Analysis

The purpose of this analysis is to determine the maximum live load reactions that will be applied to the bent. The results from this analysis will be scaled by the number of loaded lanes causing maximum responses in the bent and distributed to individual columns, for the transverse analysis.

The longitudinal analysis consists of applying various combinations of design lane and design trucks. The details can be found in AAHSTO LRFD 3.6.

6.1 Loading

In order to produce the maximum moment and reaction at interior piers, two trucks spaced at 50 feet minimum are used in the longitudinal direction per LRFD Section 3.6.1.3. The influence lines of the axial force, moment, and shear at the top and bottom column of the live loading show the effect of a two-truck loading.

6.1.1 Influence Lines

Figures below are influence lines for axial force, shear, and moment at the top of Pier 2 for a unit load moving along a girder line. The influence lines for the bottom of the pier will be exactly the same, except the moment influence will be different by an amount equal to the shear times the pier height.

To achieve the maximum compressive reaction, the lane load needs to be in spans 1 and 2, and the two trucks need to straddle between pier 2 and be as close to each other as possible. That is, the minimum headway spacing of 50 feet will maximize the axial reaction.

Maximum shears and moments occur under two conditions. First, spans 1 and 3 are loaded with the lane load and the two truck loading. The headway spacing that causes the maximum response is in the range of 180 – 200 feet. Then, a span 2 is loaded with the lane load and the two truck train. The headway spacing is at its minimum value of 50 ft.

Analytically finding the exact location and headway spacing of the trucks for the extreme force effects is possible, but hardly worth the effort. Structural analysis tools with a moving load generator, such as GTSTRUDL, can be used to quickly determine the maximum force effects.

6.2 Results

A longitudinal analysis is performed using GTSTRUDL. The details of this analysis are shown.

The result of the longitudinal analysis consists of two-truck train and lane load results. These results need to be combined to produce the complete live load response. The complete response is computed as $Q_{LL+IM} = 0.9[(IM)(\textit{Dual Truck Train}) + \textit{Lane Load}]$.

The dynamic load allowance (impact factor) is given by the LRFD specifications as 33%. Note that the dynamic load allowance need not be applied to foundation components entirely below ground level. This causes us to combine the two truck train and lane responses for cross beams and columns differently than for footings, piles, and shafts.

6.2.1 Combined Live Load Response

The tables below summarize the combined live load response. The controlling load cases are given in parentheses.

Maximum Axial

		Top of Pier	Bottom of Pier
	Axial (kips/lane)	Corresponding Moment (k-ft/lane)	Corresponding Moment (k-ft/lane)
Two-Truck Train	-117.9 (Loading case 1014)	-146.2	103.4
Lane Load	-89.1 (Loading case LS12)	-195.5	141.9
LL+IM (Column)	-221.3	-350.9	251.5
LL (Footing)	-186.3	N/A	220.8

Maximum Moment – Top of Pier

	Moment (k-ft/lane)	Corresponding Axial (kips/lane)
Two-Truck Train	-582.5 (Loading 1018)	-85.8
Lane Load	-364.2 (Loading LS2)	-49.4
LL+IM (Column)	-1025.0	-147.2

Maximum Moment – Bottom of Pier

	Moment (k-ft/lane)	Corresponding Axial (kips/lane)
Two-Truck Train	287.7 (Loading 1018)	-85.8
Lane Load	179.7 (Loading LS2)	-49.4
LL+IM (Column)	506.1	-147.2
LL+IM (Footing)	420.7	-121.7

Maximum Shear

	Shear (kips/lane)
Two-Truck Train	21.8 (Loading 1018)
Lane Load	13.6 (Loading LS2)
LL+IM (Column)	38.3
LL (Footing)	31.9

7. Transverse Analysis

Now that we have the maximum lane reactions from the longitudinal girder line analysis, we need to apply these as loads to the bent frame.

7.1 Loading

Apply the superstructure live load reactions of the longitudinal direction to substructure by placing the wheel line reactions directly to the crossbeam and varying the number and position of design lanes described in chapter 7 of the BDM.

7.2 Results

A transverse analysis is performed using GTSTRUDL. The details of this analysis are shown.

7.2.1 Cap Beam

For this example, we will look at results for three design points, the left and right face of the left-hand column, and at the mid-span of the cap beam. Note that in the analysis, the wheel line reactions were applied from the left hand side of the bent. This does not result in a symmetrical set of loadings. However, because this is a symmetrical frame we expect symmetrical results. The controlling results from the left and right hand points “A” and “B” are used.

For shear design of the crossbeam, LRFD specifications section C5.8.3.4.2 allows determination of the effects for moments and shears on the capacity of a section using the maximum factored moments and shears at the section. Hence, the results below do not show the maximum shears and corresponding moments.

The tables below summarize the results of the transverse analysis for the crossbeam. The basic results are adjusted with the multiple presence factors per LRFD Table 3.6.1.1.2-1. The controlling load cases are in parentheses.

Point A

	Shear (kips)	+Moment (k-ft)	-Moment (k-ft)
Force Effect	110.7 (Loading 1009)	0	-484.3 (1029)
Multiple Presence Factor	1.2	1.2	1.2
LL+IM	132.8	0	-581.2

Point B

	Shear (kips)	+Moment (k-ft)	-Moment (k-ft)
Force Effect	155.8 (Loading 2330)	314.3 (Loading 1522)	-650.9 (Loading 1029)
Multiple Presence Factor	1.0	1.2	1.2
LL+IM	155.8	377.2	-781.1

Point C

	Shear (kips)	+Moment (k-ft)	-Moment (k-ft)
Force Effect	87.9 (Loading 2036)	426.4 (Loading 1520)	-400.5 (Loading 1029)
Multiple Presence Factor	1.0	1.2	1.2
LL+IM	87.9	511.7	-480.6

7.2.2 Columns

The tables below show the live load results at the top and bottom of a column. The results are factored with the appropriate multiple presence factors. The controlling load cases are in parentheses.

Maximum Axial – Top and Bottom of Column

		Top of Column	**Bottom of Column**
	Axial (kips)	Corresponding Moment (k-ft)	Corresponding Moment (k-ft)
Force Effect	-347.6 (Loading 2026)	34.1	28.4
Multiple Presence Factor	1.0	1.0	1.0
LL+IM	-347.6	34.1	28.4

Maximum Moment – Top of Column

	Moment (k-ft)	Corresponding Axial (kips)
Force Effect	59.3 (Loading 1009)	-265.6
Multiple Presence Factor	1.2	1.2
LL+IM	71.2	-318.7

Maximum Moment – Bottom of Column

	Moment (k-ft)	Corresponding Axial (kips)
Force Effect	-53.6 (Loading 1029)	55.6
Multiple Presence Factor	1.2	1.2
LL+IM	-64.3	66.7

Maximum Shear

	Shear (kips)
Force Effect	-1.0 (Loading 1029)
Multiple Presence Factor	1.2
LL+IM	-1.2

7.2.3 Footings

In obtaining the footing forces of the loads from the analysis above, the linear elastic system, the principle of superposition can be used. The footing results are simply the column results scaled by the ratio of the footing load to the column load. For this case, the scale factor is 186.3÷221.3 = 0.84.

Maximum Axial – Top of Footing

	Axial (kips)	Corresponding Moment (k-ft)
LL	-292	23.9

Maximum Moment – Top of Footing

	Moment (k-ft)	Corresponding Axial (kips)
LL	-45.0	46.7

Maximum Shear – Top of Footing

	Shear (kps)
LL	-1.0

8. Combining Longitudinal and Transverse Results

To get the full set of column forces, the results from the longitudinal and transverse analyses need to be combined. Recall that the longitudinal analysis produced moments, shears, and axial load for a single loaded lane whereas the transverse analysis produced column and footing forces for multiple loaded lanes.

Before we can combine the force effects we need to determine the per column force effect from the longitudinal analysis. To do this, we look at the axial force results in transverse model to determine the lane fraction that is applied to each column.

For maximum axial load, 2 lanes at 221.3 kips/lane produce an axial force of 347.6 kips. The lane fraction carried by the column is 347.6/(2*221.3) = 0.785 (78.5%).

Mz = (-350.9 K-FT/LANE)(2 LANES)(0.785)(1.0) = -550.9 K-FT (Top of Column)

Mz = (251.5 K-FT/LANE)(2 LANES)(0.785)(1.0) = 394.9 K-FT (Bottom of Column)

Mz = (220.8 K-FT/LANE)(2 LANES)(0.785)(1.0) = 346.7 K-FT (Footing)

For maximum moment (and shear because the same loading governs) at the top of the column, 1 lane at 221.3 kips/lane produces an axial force of 318.7 kips (318.7/221.3 = 1.44), 144% of the lane reaction is carried by the column.

Mz = (-1025.0)(1.44)(1.2) = -1771.2 k-ft

Vx = (38.3)(1.44)(1.2) = 66.2 K (Column)

Vx = (31.9)(1.44)(1.2) = 55.1 K (Footing)

For maximum moment at the bottom of the column, 1 lane at 221.3 kips/lane produces an axial force of 66.7 kips (66.7/221.3 = 0.30) 30% of the lane reaction is carried by the column.

Mz = (506.1)(0.30)(1.2) = 182.2 k-ft (Column)

Mz = (420.7)(0.30)(1.2) = 151.4 k-ft (Footing)

Column

	Load Cases				
	Maximum Axial Top	Maximum Moment Bottom	Maximum Moment Top	Maximum Moment Bottom	Shear
Axial (kips)	-347.6	-347.6	- 318.7	66.7	
Mx (k-ft)	34.1	28.4	71.2	-64.3	
Mz (k-ft)	-550.9	394.9	-1771.2	182.1	
Vx (kips)					66.2
Vz (kips)					-1.2

Footing

	Load Cases		
	Maximum Axial	Maximum Moment Bottom	Shear
Axial (kips)	-292	46.7	
Mx (k-ft)	23.9	-45.0	
Mz (k-ft)	346.7	151.4	
Vx (k)			72.7
Vz (k)			-1.0

9. Skew Effects

This analysis becomes only slightly more complicated when the pier is skewed with respect to the centerline of the bridge. The results of the longitudinal analysis need to be adjusted for skew before being applied to the transverse model.

The shears and moments produced by the longitudinal analysis are in the plane of the longitudinal model. These force vectors have components that are projected into the plane of the transverse model as show in the figure below. The transverse model loading must include these forces and moments for each wheel line load. Likewise, the skew adjusted results from the longitudinal analysis need to be used when combining results from the transverse analysis.

10. Summary

This example demonstrates a method for analyzing bridge piers subjected to the LRFD HL-93 live load.

11. Longitudinal Analysis Details

The following output shows the longitudinal analysis details. In the live load generation portion of the GTSTRUDL input, you will see multiple trials for live load analysis. Each trial uses a different range of headways spacing for the dual truck train. The first trial varies the headway spacing from 180 to 205 feet. Based on this, a tighter range between 193 and 198 feet was used to get the headway spacing corresponding to the maximum loads correct to within 1 foot.

12. Transverse Analysis Details

    The following output shows the details of the transverse analysis. The interesting thing to note about the transverse analysis is the live load truck configuration. A technique of treating the wheel line reactions as a longitudinal live load is used. A two axle "truck" is created. The truck is positioned so that it is on the left edge, center, and right edge of the design lane. In order to keep the axles in the correct position, a dummy axle with a weight of 0.0001 kips was used. This dummy axial is the lead axle of the truck and it is positioned in such a way as to cause the two "real" axles to fall in the correct locations within the design lanes.

    The GTSTRUDL live load generator uses partial trucks when it is bring a truck onto or taking it off a bridge. As such, less then the full number of axles are applied to the model. For the transverse analysis, we do not want to consider the situation when only one of the two wheel lines is on the model. As such, several load cases are ignored by way of the LOAD LIST command on line76 of the output.